Journal of Southeast University (English Edition) June 2002 Vol.18 No.2 ISSN 1003—7985

Study of LDBPs Shaft Skin Friction for Piles in Cohesiove Soils

Shi Minglei* Deng Xuejun Liu Songyu

(College of Traffic and Transportation Engineering, Southeast University, Nanjing 210096, China)

Abstract: The methodology of predicting pile shaft skin ultimate friction has been studied in a systematic way. In the light of that, the analysis of the pile shaft resistance for bored and cast-in-situ piles in cohesive soils was carried out thoroughly in the basis of field performance data of 10 fully instrumented large diameter bored piles (LDBPs) used as the bridge foundation. The undrained strength index μ in term of cohesive soils was brought forward in allusion to the cohesive soils in the consistence-plastic state, and can effectively combine the friction angle and the cohesion of cohesive soils in undrained condition. And that the classical "α method" was modified much in effect to predict the pile shaft skin friction of LDBPs in cohesive soils. Furthermore, the approach of standard penetration test (SPT) N value used to estimate the pile shaft skin ultimate friction was analyzed, and the calculating formulae were established for LDBPs in clay and silt-clay respectively.

Key words: large diameter bored piles, pile shaft skin friction, blow count of standard penetration test

For the prediction of the axial pile bearing capacity, there are many approaches. Each method has its benefits and drawbacks and none is universally accepted. The methodologies to estimate the axial load capacity are generally grouped into three broad categories: full-scale load tests, the static approaches and the dynamic methods. A relative more reliable design will be attained based on the results from the full-scale static load tests. However, the load tests are costly and time consuming. Based on pile driving dynamics, the procedures in the dynamic method make use of the pile-ground dynamic interaction model and apply the stress-wave matching techniques. In this group, it is the most difficult that how the soil-pile interaction model is defined effectively by empirical method.

The other approaches to the calculation of the ultimate load capacity of a single piles, known as the "static" approach, which uses the normal soil-mechanics method to calculate the load capacity on the basis of measured soil properties obtained from laboratory or in-situ tests, will be discussed in this paper. In attempt to summarize and compare the various predicting methods in the static methods, one faced with the vast spectrum of methods and approaches. In general, the methods can be subdivided into three categories[1]:

1) Design by tables, based on soil classification data;

2) Design by static bearing capacity formulae, using basic soil parameters;

3) Design by semi-empirical correlations between the soil test and the pile behavior.

1 Design on Basis of Laboratory Test Data

For piles in cohesive soils, the undrained load capacity is generally taken to be the critical value unless the cohesive soils are highly overconsoildated[2]. Based on the undrained conditions generally prevailing in the soil near the pile shaft, the α-method is usually taken to calculate the bearing capacity of a single pile in cohesive soils. If the soils is saturated, the undrained friction angle φ_u of that is zero, and the friction angle φ_a between pile and soil may also be taken as zero, so that the pile shaft resistance f_s can be evaluated as

$$f_s = c_a = \alpha \cdot c_u \tag{1}$$

where α is adhesoin factor, $\alpha = 0.35 - 0.80$ and $f_s \leqslant$ 80 kPa in general; c_u is undrained cohesion of clay; c_a is undrained pile-soil adhesion.

It is very clear that the adhesoin factor α is the key part in Eq.(1) used to predict the bearing capacity for single pile in clay. In the strength range of interest, Tomlinson's relation (2) for α can be expressed as[3]

$$\alpha = 1.0 + 0.5\left(\frac{c_u}{50} - 0.5\right) \tag{2}$$

The typical relationship between α and c_u, based on the summary provided by McClleland[2], are shown that the value of c_u decreases with the value of c_u increasing, and the lower limit of $\alpha = 0.35$ for the value of $c_u > 200$ kPa. It is generally agreed that for

Received 2001-10-29.

* Born in 1962, male, associate professor.

soft clays($c_u < 24$ kPa), the value of $\alpha = 1.0$ (or even greater).

2 Design on Basis of In-Situ Test Data

In-situ soil tests such as CPT, PMT, and SPT are widely used in various countries to predict the axial bearing capacity of a single pile, and that have attracted many researchers over decades resulting in different solutions, known as one category of the static approaches in term of data from field tests. In Tab.1, a schematic list of the in-situ tests commonly used in pile design is given.

Tab.1 Overview of field explorations used in piles

Common investigation methods	Acronym	Remarks
Static cone penetrometer testing	CPT/CPTU	Extensive
Pressuremeter test	PMT(MPM, SBP, PIP)	Frequently
Vane		In soft soils only
Dilatometer test	DMT	In the midst of research
Geophysical tests	SASW, CH	Rare exception, but be of significant

The semi-empirical approaches, starting from these in-situ tests, involves three categories. The first approach is in particular followed the CPT, which by itself measures separately point resistance and pile shaft friction for a "small diameter steel pile of displacement type". The methodology based on the field explorations as a direct model, is dominating in Belgium and the Netherlands, and also widely used in other countries such as France, Italy, Poland and China, etc. The second method considers the field explorations in direct correlation with the pile's behavior, e.g. standard penetration test data and pressuremeter test data. The correlation might be founded on theoretical and analytical consideration such as cavity expansion model for both PMT probe and driven piles. In practice, however, the correlation is mostly expressed as simple empirical formulae by database regress. In the third approach, in-situ tests data only serve as a basis for deduction of other parameters in pile design to allow the use of design methods based on these alternative indexes.

In practice, the methodology in accordance with categories the first and the second is rather similar in various countries, and can be generally summarized in the following formulae for total ultimate pile resistance Q_u, the unit shaft skin friction f_s and the unit end bearing resistance q_b.

$$Q_u = Q_b + Q_s = \sum f_{si} \cdot A_i + A_b \cdot q_b \tag{3}$$

where

$$q_b = \alpha_b \cdot q_{ce} (\text{ or } p_{le} \text{ or } N_e) \tag{4}$$

$$f_{si} = \alpha_s \cdot F_{si} (\text{ or } q_{ci} \text{ or } p_{1i} \text{ or } N_i) \tag{5}$$

where α_b and α_s are empirical factors taking account of the pile installation, the soil type and the nature, and roughness of pile shaft's material; q_c is the special penetration resistance from the CPT; F_s is the CPT total side friction increment on the CPT rod; N is the SPT blow count; p_1 is the resistance increment from the PMT.

3 Determination of Pile Shaft Resistance for LDBPs in Cohesive Soils

On the basis of the subjects*, the performance of LDBPs in cohesive soils, used as deep foundation of heavy bridge structures, has been researched in a systematic way. The shaft skin resistance was investigated based on the database of 10 vertical static loaded tests, in which the LDBPs of a diameter 1.0 m, generally varying from 37 to 45 m in shaft length, had been instrumented with steel bar sensors in advance, and, therefore, the inner stress along the pile shaft could be tested during loading procedure. The piles were bored in layered soils, and the geotechnical profiles were characterised by cohesion soils, rare exceptionaly which were separated by silty sands and in general varies slightly in differ history cases of the freeway projects. The pile shaft skin resistance for the pile* in cohesive soils from static load tests of instrumented pile are shown in Tab.2 and Tab.3.

3.1 The α-method modified

The α-method is mostly taken to estimate the pile shaft skin resistance for a single pile in cohesive soils. Of course, this procedure is suitable for LDBPs in soils as well. In the cases of the freeway projects, the deposit upon bedrock involves primarily clays and silt-clays, in general, which naturally exists in plastic state or consistence-plastic state. The field investigation has founded clearly that the values of φ_u are not zero commonly for clays and silt-clays in the cases (see Tab.2 and Tab.3), but rare layers of saturated soft silt-clay, as the only exception to that, and their values of φ_u approach zero. The values of φ_u for certain silt clays(shown in Tab.3) achieve the high value so much as prevailing in the soils strength rela-

* Special testing report of the bearing capacity and the inner stress for LDBPs used in bridge foundations for the Xiyi freeway(Feb 2001, Southeast University); Special testing report of the bearing capacity and the inner stress for LDBPs used in bridge foundations for the Xichen freeway(April 1997, Southeast University).

Tab.2 Shaft resistance f_s for pile in clay

f_s/kPa	φ_u/(°)	c_u/kPa	μ	SPT-N	f_s/kPa	φ_u/(°)	c_u/kPa	μ	SPT-N
53.36	11.0	91.5	28.37	15.1	72.00	11.4	65.0	38.5	11.7
71.67	11.4	65.0	24.17	11.7	82.00	16.1	65.3	38.5	10.8
81.25	16.1	65.3	27.59	10.8	80.00	11.8	53.0	43.0	14.0
65.21	10.6	53.0	21.37	15.9	57.00	10.6	53.0	38.5	15.9
49.34	10.6	53.0	21.37	15.9	92.00	15.2	37.0	47.0	14.2
79.86	11.8	53.0	22.06	14.0	41.00	14.8	30.0	38.5	5.9
91.78	12.6	78.0	28.89	13.0	92.00	12.6	87.0	43.0	13.0
53.48	14.0	44.0	21.38	14.2	63.00	11.0	91.5	40.8	10.9
84.40	15.2	37.0	20.25	11.7	53.00	14.0	44.0	43.0	14.0

Tab.3 Shaft resistance f_s for pile in silt-clay

f_s/kPa	φ_u/(°)	c_u/kPa	μ	SPT-N	f_s/kPa	φ_u/(°)	c_u/kPa	μ	SPT-N
35.44	8.5	15.0	10.76	4.7	81.89	25.8	12.0	16.22	14.1
41.25	19.5	10.0	11.99	4.4	35.53	5.0	16.6	10.38	3.2
48.07	19.5	10.0	11.99	4.0	59.11	13.9	25.0	16.06	8.1
39.95	19.5	10.0	11.99	3.6	53.48	14.8	25.8	16.72	7.0
44.91	2.5	20.4	10.84	2.9	81.03	19.8	35.4	22.80	11.5
95.90	15.0	34.2	19.38	11.0	92.26	19.8	35.4	22.80	11.0
85.31	15.0	34.2	19.38	11.0	76.45	10.9	28.0	15.65	7.0
72.24	15.0	34.2	19.38	11.0	65.58	20.6	35.1	23.22	11.6
75.29	13.0	51.0	22.36	9.2	83.00	13.0	51.0	22.36	8.7
88.10	13.0	51.0	22.36	9.5	35.00	5.0	16.6	10.38	3.2
86.97	13.0	51.0	22.36	10.0	65.00	23.2	22.0	20.06	10.2
83.66	5.5	42.5	16.81	5.9	42.00	2.5	20.4	10.84	3.0
77.53	13.3	41.3	20.32	9.4	78.00	13.3	41.3	20.32	8.0
103.80	13.3	41.3	20.32	7.9	59.00	13.9	25.0	16.06	8.0
118.99	16.4	55.9	25.79	13.2	87.00	19.8	35.4	22.80	11.0
124.32	16.4	55.9	25.79	13.2	64.00	11.1	32.0	16.82	5.0
96.10	16.4	55.9	25.79	13.2	53.00	14.8	25.8	16.72	7.0
70.58	16.4	55.9	25.79	13.2	101.00	16.4	55.9	25.79	13.2
64.46	23.2	22.0	20.06	11.1	89.00	15.0	34.2	19.38	11.0
64.33	11.1	32.0	16.82	5.5	66.0	20.6	35.1	23.22	11.6
					41.00	19.5	10.0	11.99	4.2

tively. Accordingly it is irrational that the pile shaft skin friction f_s is derived from the unique c_u. The α-method modified to coupling the φ_u and c_u for piles in cohesive soils was given by

$$f_s = \alpha\mu \tag{6}$$

$$\mu = \left[c_u \tan\left(45° + \frac{\varphi_u}{2}\right) \right]^{0.5} \tag{7a}$$

$$\mu = \left\{ c_u \left[e^{\pi\tan\varphi_u} \tan^2\left(45° + \frac{\varphi_u}{2}\right) - 1 \right] \tan^{-1}\varphi_u \right\}^{0.5} \tag{7b}$$

where μ is a composite parameter, called as undrained strength index.

The undrained strength index μ in term of cohesive soils combines its φ_u and c_u. For calculating μ, Eq.(7a) was established in term of the unconfined compression strength, and Eq.(7b) was constituted in the light of the classical plasticity solution for the bearing capacity of surface footing established by Prandlt[2]. On the database result from 10 vertical static loaded tests, the α-method modified has been established to predict f_s for LDBPs in cohesive soils, and the results derived from that are shown in Tab.4.

Tab.4 gives a summary of the adhesion factors α for both the clay and the silt-clay in the cases, as well as the statistical characteristics of α. Now that the coefficient of variation derived in Eq. (7b) is much less than that in Eq.(7a), Eq.(7b) is more recommendable

Tab.4 Summary of the value of α in term of μ

Soil classification	Number of specimens	Eq.(7a)		Eq.(7b)	
		Adhesion factor α	Coefficient of variation	Adhesion factor α	Coefficient of variation
Clay	18	8.32	0.515	2.98	0.159
Silt-clay	42	11.39	0.520	3.96	0.223

in relative and adopted indeed in this paper. Moreover, the α was founded to be approximate horizontal linear function of the index μ in term of Eq.(7b), and the correlation has been plotted in Fig.1 and Fig.2 for clays and silt clays, respectively. The value of α decreases slightly with the μ increasing, and the value of it proposed for applying in practice are shown in Tab.5.

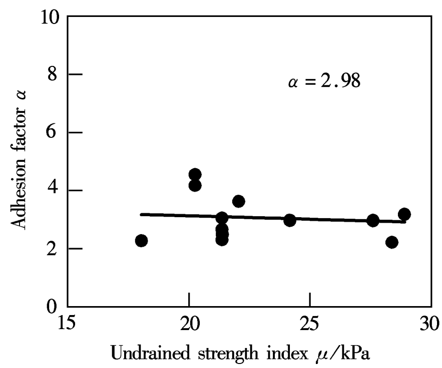

Fig.1 The adhesion factor as a function of the undrained strength index of clay

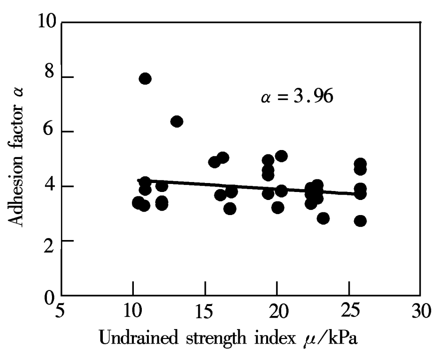

Fig.2 The adhesion factor as a function of the undrained strength index of silt-clays

Tab.5 The value of α in term of μ

Classification	Clay	Silt-clay
Range	2.0 - 4.0	3.0 - 5.0
Mean	3.0	4.0

For μ, known as a composite parameter for cohesive soils, coupling φ_u with c_u effectively, the modified approach in that the pile shaft resistance, especially while a pile in cohesive soils in which the φ_u prevails over c_u, is derived from the index μ is much more rational than the classical α-method.

3.2 Correlation of SPT blow count N with pile shaft resistance

For piles in cohesive soils, the methodology that the pile shaft resistance is derived from field tests (CPT, PMT) is of great significant and has been studied as a subject worthy of many researchers over decades. The SPT is one simple situ exploration method which has been commonly used in granular soils[1,4], and, however, up to date this method has already been utilized to study the behaviour of pile in cohesive soils[7,8]. The correlation of SPT blow count N with f_s has been derived from the database of 10 vertical static loaded tests for LDBPs in the cases.

Tab.6 Shaft resistance in cohesive soils predicted by SPT

Classification	Specimens	Eq.(8)	
		Coefficient of regression α_s	Coefficient of variation $V_{\alpha s}$
Clay	18	5.56	0.409
Silt clay	42	7.38	1.227

Tab.7 Shaft resistance in silt-clays predicted by SPT

Item	Specimens	Blow count N unmodified	Blow count N' modified
Coefficient c_1		36.25	31.89
Coefficient c_2	42	3.16	4.72
Standard deviation σ		5.57	6.16

Tab.6 shows that the approach by which f_s is derived from the simple Eq.(8) is of enough accurate for piles in clay, and, however, that of not for piles in silt-clays(see Tab.6). Eq. (8) for piles in clays is given as

$$f_s = \alpha_s N = 5.56N \tag{8}$$

Therefore it is a comparatively simple step from Eq.(8) to Eq.(9), which is founded to predict f_s for piles in silt-clays very effectively, and besides, Tab.7 proves that the value of f_s derived from SPT N value more accurate in contrast with the SPT N' value modified with length of the drill pipe. So that the empirical formula for piles in silt clays can be expressed in term of N as

$$f_s = c_1 + c_2 N = 36.25 + 3.16N \tag{9}$$

The distribution of f_s by SPT N value is shown in Fig.3 and similarly the relationship between f_s and the modified SPT N' value can be plotted in Fig.4 as well.

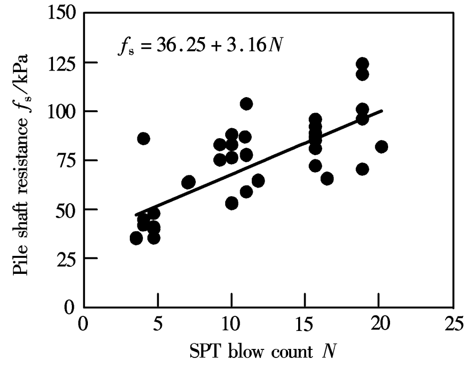

Fig.3 The pile shaft skin friction as a function of the SPT count N

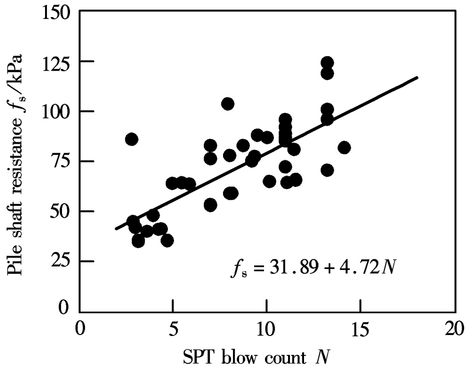

Fig.4 The pile shaft skin friction as a function of the SPT count N'

4 Conclusions

In this study, an attempt has been made to predict the shaft resistance for the LDBPs in cohesive soils in this formation using the modified α-method and an empirical approach in term of SPT N value. In the process of analysis based on the database of the case histories in the previous paragraphs, the following conclusions are drawn:

1) The undrained strength index μ (see Eq. (7b))constituted in this paper to coupling φ_u with c_u can be much effectively used in the α-method to predict the shaft resistance for piles in cohesive soils, particularly which exists in a consistence-plastic state or more consistence state.

2) The correlation of pile shaft resistance with the SPT N (see Eq. (8) and Eq. (9))derived from estimation of the shaft resistance of the piles in cohesive soils is of great significant in practice and of the value referenced in future.

References

[1] Cock E. Design of axially load bored piles — European codes, practice[A]. In: Van Impe & Heageman, eds. *Deep Foundation on Bore and Auger Piles* [C]. Balkema, Rotterdam, 1998 63 – 74.

[2] Poulos H G, Davis E H. *Pile foundation analysis and design* [M]. New York: John Wiley and Sons, 1980. 18 – 51.

[3] Richard J Finno. *Predicted and observed axial behavior of piles* [M]. New York: Geotechnical Special Publication No. 23 published by the American Society of Civil Engineers, 1989. 97 – 106.

[4] Alessandro Mandolini. Design of axially loaded piles-Italian practice [A]. In: De Cock & Legrand, eds. *Design of Axially Loaded Piles— European Practice*[C]. Balkema, Rotterdam, 1997. 219 – 242.

[5] Findlay J D, Brooks J N, Mure J N, et al. Design of axially loaded piles-United Kingdom practice[A]. In: De Cock & Legrand, eds. *Design of Axially Loaded Piles— European Practice*[C]. Balkema, Rotterdam, 1997. 353 – 376.

[6] Burland J B. Shaft friction of piles in clay—A simple fundamental approach [J]. *Ground Eng*, 1973, **6**(3): 30 – 21.

[7] Balakrishnan E G, Balasubramaniam A S, Noppadol Phienwej. Load deformation analysis of bored piles in residual weathered formation[J]. *Journal of Geotechnical and Geoenvironmental Engineering February*, 1999. 122 – 131.

[8] Kuwabara E, and Tanaka M. Statistical analysis on shaft friction of vertically loaded bored piles[A]. In: Van Impe & Haegeman, eds. *Deep Foundations on Bored Auger Piles* [C]. Balkema, 1998. 221 – 228.

粘性土中 LDBPs 桩侧极限摩阻力研究

石名磊 邓学钧 刘松玉

(东南大学交通学院,南京 210096)

摘　要　根据 10 根试桩静载荷试验和桩身应力测试结果,在对桩侧极限摩阻力预测技术系统分析的基础上,研究了粘性土中 LDBPs 桩侧极限摩阻力的预测方法和指标确定.针对硬塑状态的粘性土,提出了不排水强度指数 μ 的概念,综合了粘性土不排水条件下内摩擦角和粘聚力.基于这一概念,对传统的“α 法”进行了扩展,使之更加有效地应用于粘性土中 LDBPs 桩侧摩阻力的预测.同时还对 SPT 锤击数 N 预测粘性土中 LDBPs 的桩侧摩阻力进行了统计分析,并对粘土和亚粘土分别提出了相应的回归公式.

关键词　大直径钻孔灌注桩, 桩侧摩阻力, 标准贯入击数

中图分类号　U443.154